CLASS TEST

S.No. : 01 IG_RCC_A+B_10522

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REINFORCED CEMENT CONCRETE

CIVIL ENGINEERING

Date of Test : 10/05/2022

ANSWER KEY

1.	(a)	7.	(a)	13.	(c)	19.	(b)	25.	(c)
2.	(a)	8.	(c)	14.	(b)	20.	(a)	26.	(b)
3.	(b)	9.	(b)	15.	(d)	21.	(b)	27.	(a)
4.	(d)	10.	(b)	16.	(a)	22.	(a)	28.	(d)
5.	(d)	11.	(c)	17.	(a)	23.	(a)	29.	(d)
6.	(d)	12.	(a)	18.	(a)	24.	(b)	30.	(b)

1. (a)

$$\frac{A_{sv}}{b \cdot S_V} \geq \frac{0.4}{0.87 f_y}$$

$$\Rightarrow \quad A_{sv} \geq \frac{0.4 \times b \times S_V}{0.87 \times f_y}$$

$$\Rightarrow \quad A_{sv} \geq \frac{0.4 \times 400 \times 100}{0.87 \times 415} = 44.3 \text{ mm}^2 \simeq 45 \text{ mm}^2$$

$\therefore$ Minimum shear reinforcement = 45 mm^2

2. (a)

Pitch of helical turns shall not be more than

(i) 75 mm

(ii) one-sixth of the core diameter of column $\frac{480}{6} = 80$ mm

} = 75 mm

Pitch of helical turns shall not be less than

(i) 25 mm

(ii) 3 times the diameter of steel bar forming the helix

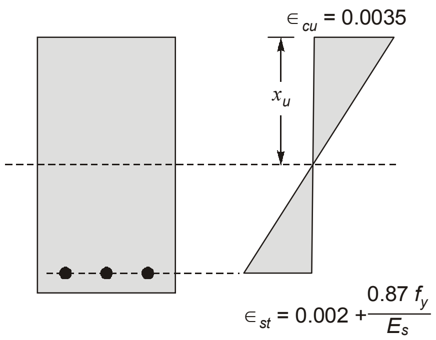
So, maximum pitch is 75 mm.

3. (b)

Long term modulus of elasticity of concrete,

$$E_{cl} = \frac{E_C}{1+\theta} = \frac{5000\sqrt{f_{ck}}}{(1+\theta)} \text{ MPa}$$

4. (d)



For Fe415,

$$\in_{st} = 0.002 + \frac{0.87 \times 415}{2 \times 10^5} = 3.805 \times 10^{-3}$$

5. (d)

Design load for collapse:

(i) 1.5 DL + 1.5 LL = 1.5 × 120 + 1.5 × 200 = 480 kN/m

(ii) 1.5 DL + 1.5 WL = 1.5 × 120 + 1.5 × 25 = 217.5 kN/m

(iii) 1.2 DL + 1.2 LL + 1.2 EL = 1.2 × 120 + 1.2 × 200 + 1.2 × 25 = 414 kN/m

So, maximum of these three values will be the design load for collapse condition = 480 kN/m

6. (d)

$$p_{tlim} = 41.61\left(\frac{f_{ck}}{f_y}\right)\left(\frac{x_{u\lim}}{d}\right)$$

$$= 41.61\left(\frac{35}{415}\right)(0.48) = 1.68\%$$

8. (c)
The concept of load balancing is applied for determinate structures. Concordant cable profile is used for indeterminate structures.

10. (b)

$$\text{SF along edge } BC = \frac{wl_{BC}}{4} = \frac{12 \times 4}{4} = 12 \text{ kN}$$

11. (c)

$$\text{Development length,} \quad l_d = \frac{\phi\sigma_{st}}{4\tau_{bd}} = \frac{25 \times 0.87 \times 415}{4 \times (1.6 \times 1.4)} = 1008 \text{ mm}$$

For 90° bend, l_d is reduced by 8ϕ.

$$l_d = 1008 - 8 \times 25 = 808 \text{ mm}$$

13. (c)

$$\text{Area of steel, } A_{st} = 3 \times \frac{\pi}{4} \times 20^2 = 942.5 \text{ mm}^2$$

For Fe 415, $x_{u,max} = 0.48\, d$

$\Rightarrow$ $x_{u,max} = 0.48 \times 400 \text{ mm} = 192 \text{ mm}$

Now, Compressive force, $C = 0.36\, f_{ck}\, Bx_u$

Tensile force, $T = 0.87\, f_y\, A_{st}$

Since, $C = T$

$$\Rightarrow \quad x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} B} = 151.24 \text{mm}$$

As $x_u < x_{u,max}$, the section is under-reinforced.

14. (b)
Let the dip of the cable be 'h'.

Upward pressure provided by the parabolic cable $= \dfrac{8Ph}{L^2}$

In order that this upward pressure may fully balance the external loading.

$$\Rightarrow \quad \frac{8Ph}{L^2} = w$$

$$\Rightarrow \quad \frac{8 \times 2500 \times h}{(10)^2} = 40$$

$$\Rightarrow \quad h = 0.2 \text{ m} = 200 \text{ mm}$$

15. (d)
If A is the cross-sectional area of the column, A_{st} is the area of steel, A_c is area of concrete, m is modular ratio and f_c is stress in concrete, then

$$A_{st} = \frac{1}{100} \times A = 0.01 \text{ A}$$

$$A_c = A - A_{st} = A - 0.01 \text{ A} = 0.99 \text{ A}$$

$$\therefore \quad \frac{P_s}{P_c} = \frac{m A_{st} f_c}{A_c f_c}$$

$$= \frac{10 \times 0.01 A \times f_c}{0.99 A \times f_c} \times 100 = 10.10\%$$

16. (a)

$$\text{Area of footing} = 2 \times 3 = 6\text{m}^2$$

$$\text{Section modulus, } Z = \frac{b \times d^2}{6} = \frac{2 \times 3^2}{6} = 3\text{m}^3$$

$$\text{Stress, } \sigma = \frac{P}{A} \pm \frac{M}{Z}$$

$$95 = \frac{P}{A} + \frac{M}{Z} \quad \ldots(i)$$

$$55 = \frac{P}{A} - \frac{M}{Z} \quad \ldots(ii)$$

From (*i*) and (*ii*), we get

$$\Rightarrow \quad 40 = 2 \times \frac{M}{Z}$$

$$\Rightarrow \quad 40 = 2 \times \frac{M}{3}$$

$$\Rightarrow \quad M = 60 \text{ kNm}$$

17. (a)

$$DL = 0.32 \times 0.4 \times 25 = 3.2 \text{ kN/m}$$

$$LL = 10 \text{ kN/m}$$

$$TL = 10 + 3.2 = 13.2 \text{ kN/m} = W$$

$$P = 1000 \times 100 = 100 \text{ kN}$$

$$\theta_1 = \frac{PeL}{2E_cI} = \frac{100 \times 10^3 \times 80 \times 6500}{2 \times 25 \times \frac{320 \times 400^3}{12} \times 1000} = 6.09375 \times 10^{-4} \text{ radian}$$

$$\theta_2 = \frac{WL^3}{24E_cI} = \frac{13.2 \times 6500^3}{24 \times 25 \times \frac{320 \times 400^3}{12} \times 1000} = 3.54 \times 10^{-3} \text{ radian}$$

$$\text{Net rotation, } \theta = \theta_2 - \theta_1 = 29.306 \times 10^{-4} \text{ radian}$$

$$\text{Net elongation} = 2e\theta = 2 \times 80 \times 29.306 \times 10^{-4} = 0.468 \text{ mm} \simeq 0.47 \text{ mm}$$

18. (a)

$$\text{Strain due to shrinkage} = \frac{2 \times 10^{-4}}{\log_{10}(t+2)} = 2 \times 10^{-4}$$

$$\therefore \quad \text{Loss of prestress, } \Delta\sigma = 2 \times 10^{-4} \times E_s$$

$$= 2 \times 10^{-4} \times 2 \times 10^5 = 40 \text{ N/mm}^2$$

$$\therefore \quad \text{Percentage of prestress loss} = \frac{40 \times 100}{500} = 8\%$$

19. (b)

Stress in concrete at the level of tendon,

$$f_c = \frac{P}{A} + \frac{Pe^2}{I}$$

$$= \frac{150 \times 10^3}{120 \times 200} + \frac{150 \times 20^2 \times 12 \times 10^3}{120 \times 200^3} = 7 \text{ MPa}$$

Loss of prestress due to elastic deformation,

$$\Delta f_s = \frac{E_s}{E_c} \times f_c = \frac{2.1 \times 10^5}{3.0 \times 10^4} \times 7 = 49 \text{ MPa}$$

Total stress in steel, $f_s = \frac{P}{A_s} = \frac{150 \times 10^3}{187.5} = 800$ MPa

$\therefore$ Percentage loss of stress $= \frac{\Delta f_s}{f_s} \times 100 = \frac{49}{800} \times 100 = 6.125\% \simeq 6.13\%$

21. (b)

One-way shear stress, $\tau_v = \frac{V_u}{Bd}$

V_u = Factored vertical shear force at critical section

Net factored pressure on the footing $= \frac{400 \times 1000}{2000 \times 2000} = 0.1\ \text{N/mm}^2$

As the critical section is at a distance d (250 mm) from the face of the column,

$$V_u = 0.1 \times \left(\frac{2000 - 300}{2} - 250\right) \times 2000$$

$$= 0.1 \times 600 \times 2000 = 120 \times 10^3\ \text{N}$$

Now, $\tau_v = \frac{120 \times 10^3}{2000 \times 250} = 0.24\ \text{N/mm}^2$

22. (a)

$$e_{min} = \text{Max. of } \frac{l}{500} + \frac{D}{30} \text{ or } 20 \text{ mm}$$

$$= \frac{3000}{500} + \frac{450}{30} = 21\ \text{mm}$$

And, $0.05D = 0.05 \times 450 = 22.5$ mm

As $e_{min} < 0.05D$, It is axially loaded column

Now, $P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

$$= 0.4 \times 20 \times \left(\frac{\pi}{4} \times 450^2 - A_{sc}\right) + 0.67 \times 415 \times A_{sc}$$

Putting, $A_{sc} = 6 \times \frac{\pi}{4} \times 20^2 \simeq 1885\ \text{mm}^2$

P_u = 1781.39 kN (Factored load)

Maximum service axial load $= \frac{1781.39}{1.5} = 1187.59$ kN

23. (a)

Effective depth, $d = 500 - 30 - \frac{20}{2} = 460$ mm

Since, $C = T$

$\Rightarrow$ $0.36 f_{ck} b x_u = 0.87 f_y A_{st}$

$$\Rightarrow \quad x_u = \frac{0.87 \times 415 \times 3 \times \frac{\pi}{4} \times 20^2}{0.36 \times 20 \times 250} = 189\ \text{mm}$$

Now, $x_{u,lim} = 0.48 \times 460 = 220.8$ mm

As $x_u < x_{u,lim}$, Section is under-reinforced.

$$M_{u,lim} = 0.36 f_{ck} b x_u (d - 0.42 x_u)$$

$$= 0.36 \times 20 \times 250 \times 189 (460 - 0.42 \times 189) = 129.48 \text{ kNm} \simeq 130 \text{ kNm}$$

24. (b)

$$C_{uc} + C_{us} = T_u$$

$$\Rightarrow \quad 0.36\, f_{ck}\, bx_u + (f_{sc} - 0.45\, f_{ck})A_{sc} = 0.87\, f_y\, A_{st}$$

$$\Rightarrow \quad x_u = \frac{0.87 \times 250 \times A_{st} - (f_{sc} - 0.45 \times 20)A_{sc}}{0.36 \times 20 \times 280}$$

$$A_{st} = 5 \times \frac{\pi}{4} \times 24^2 = 2262\,\text{mm}^2$$

$$A_{sc} = 3 \times \frac{\pi}{4} \times 16^2 = 603.2\,\text{mm}^2$$

Assuming, $f_{sc} = 0.87\, f_y = 0.87 \times 250 = 217.5\ \text{N/mm}^2$

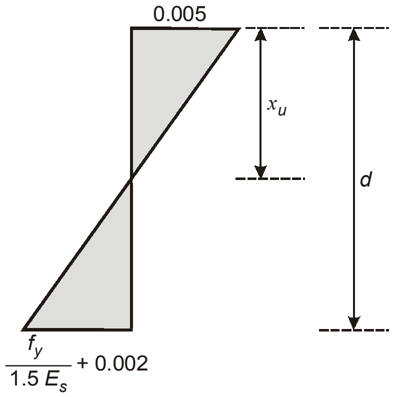
$\Rightarrow$ $x_u = 181.65\ \text{mm}$

Also, $x_{u,\,max} = 0.53d = 0.53 \times 450 = 238.5\ \text{mm}$ $(\because\ \text{Fe250})$

25. (c)

According to new **IS 456**

Linear strain diagram



$$\frac{0.005}{x_u} = \frac{\frac{f_y}{1.5 \times 2 \times 10^5} + 0.002}{d - x_u}\ \text{m}$$

$$\frac{x_u}{d} = 0.596$$

Limiting depth of neutral axis using Fe 415 according to IS 456:2000 = 0.48 d

∴ Difference in the limiting depth of neutral axis = 0.596 d – 0.48 d = 0.116 d ≈ 0.12 d

26. (b)

$$M_{eq} = M + M_T = M + \frac{T}{1.7}\left(1 + \frac{D}{b}\right) = 50 + \frac{20}{1.7}\left(1 + \frac{400}{400}\right) = 73.53\ \text{kNm}$$

$$V_{eq} = V + 1.6\frac{T}{B} = 25 + \frac{1.6 \times 20}{0.4} = 105\ \text{kN}$$

$$\frac{M_{eq}}{V_{eq}} = \frac{73.53}{105} = 700.29\,\text{mm}$$

27. (a)

As per Clause 29.2 of **IS 456:2000**

$$(LA)_{\text{SSB}} = 0.2\,(l + 2D) \qquad \left(\text{For } 1.0 \le \frac{l}{D} < 2\right)$$

$$= 0.2\,(6 + 2 \times 5) = 3.2\ \text{m}$$

$$(LA)_{CB} = 0.2\,(l + 1.5D) \qquad \left(\text{For } 1.0 \le \frac{l}{D} < 2.5\right)$$

$$= 0.2\,(6 + 1.5 \times 5) = 2.7 \text{ m}$$

29. (d)

Factored shear force $= 1.5 \times 110 = 165$ kN

Effective depth $= 500 - 35 = 465$ mm

$$A_{st} = 2 \times \frac{\pi}{4} \times (20)^2 = 628.32 \text{ mm}^2$$

Characteristic strength of steel, $f_y = 415$ N/mm^2

Moment of Resistance, $M_u = 0.87\, f_y A_{st}\,(d - 0.42x_u)$

Depth of neutral axis, $x_u = \dfrac{0.87\, f_y A_{st}}{0.36\, f_{ck} b} = \dfrac{0.87 \times 415 \times 628.32}{0.36 \times 20 \times 250}$

$= 126.03$ mm $< x_{u,\lim}$ ($x_{u,\lim} = 0.48d = 0.48 \times 465 = 223.2$ mm)

$$M_u = 0.87 \times 415 \times 2 \times \frac{\pi}{4} \times (20)^2 \times (465 - 0.42 \times 126.03)$$

$= 93.48 \times 10^6$ N-mm $= 93.48$ kNm

The anchorage value of a standard U-type hook is equal to 16 ϕ.

($\because$ For every 45° bend, anchorage value is 4ϕ)

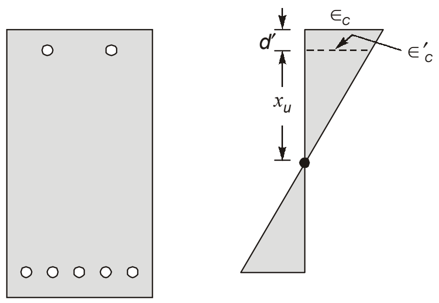
$$L_0 = 16\,\phi = 16 \times 20 = 320 \text{ mm}$$

According to IS 456, $L_d \le \dfrac{1.3 M_1}{V} + L_0$

$$\le \frac{1.3 \times 93.48 \times 10^6}{165 \times 1000} + 320$$

$$L_d \le 1056.51 \text{ mm}$$

30. (b)



Maximum strain in concrete $= 0.0035$

$$\frac{\epsilon_c'}{x_u - d'} = \frac{\epsilon_c}{x_u}$$

$$\epsilon_c' = \epsilon_c\left(\frac{x_u - d'}{x_u}\right) = 0.0035\left(\frac{x_u - d'}{x_u}\right)$$

$$= 0.0035\left(\frac{150 - 40}{150}\right) = 2.56 \times 10^{-3}$$

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